NATIONAL INSTITUTE OF TECHNOLOGY ROURKELA

SUBMISSION OF PROJECT REPORT

ON

DYNAMIC ANALYSIS OF A MULTISTOREY FRAME

Under the guidance of Prof
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SUBMITTED
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CERTIFICATION

This is to certify that Alapan Bhowmik, Roll no 10501019 has done his project work in his final year B.Tech degree course on the subject of “Dynamic Analysis of a multi-storey frame” under the guidance of Prof A.V.Asha, dept of Civil Engg, NIT ROURKELA.

Forwarded
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ACKNOWLEDGEMENT

I would like to express my deep gratitude to my project guide Prof A.V.Asha for her able guidance. It is only for her constant suggestions that I have been able to finish my project work.

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CONTENTS

1. **INTRODUCTION**

In every aspect of human civilisation we needed structures to live in or to get what we need. But it is not only building structures but to build efficient structures so that it can fulfil the main purpose for what it was made for. Here comes the role of civil engineering and more precisely the role of analysis of structure. There are many classical methods to solve design problem, and with time new softwares also coming into play. Here in this project work recent fem based software named staad pro has been used. Few standard problems also have been solved to show how staad pro can be used in different cases. It was not possible to venture into every aspect of analysis. Static analysis, Seismic analysis and natural frequency have been done using Staad pro. These typical problems have been solved using basic concept of loading, analysis, condition as per is code. These basic techniques may be found useful for further analysis of problems. Staad pro gives more precise and accurate results than manual techniques.

2. **Staad pro: An overview**

Staad pro is software which is based on the techniques of finite element method. It is one of the most popular software available now days. The software mainly does design works. The main steps in staad pro doing problems are

Creating the geometry using different methods

Defining the Cross-Sections of Beams, Columns, Plates

Defining the Constants, Specifications, and Supports

Defining the Load Systems

Analyzing your Model using the appropriate
Analysis method

Reviewing the Analysis Results

Performing Concrete Design

Performing Steel Design

3. STATIC ANALYSIS

3.1 TRUSS:
A plane truss is defined as a system of bars, all lying in one plane and joined together at their ends in a such way as to form a rigid frame work. When a truss is supported at vertical plane and is subjected to loads at joints, reactions will be produced at the supports. To balance the external forces internal axial forces will be produced at the members. Determination of these forces is known as analysis of truss.

3.2 BASIC EQUILIBRIUM EQUATIOS:

Any body initially at rest and that continues to remain at rest as loads are applied is said to be in state of static equilibrium. When any motion occurs the body is said to be in dynamic equilibrium. A single body in two dimensional space will be in static equilibrium, if forces in x direction,y direction, and moment about a point is summed up to zero.

3.3 EQUILIBRIUM, COMPATIBILITY AND FORCE DISPLACEMENT RELATION:

The complete analysis of truss will require the use of three principles of

1. Equilibrium.

2. Compatibility
.
3. Force Displacement relations.

In the cases of two dimensional analysis the three basic equation of equilibrium are to be satisfied. All the forces in x direction, y direction and moment about a particular point will summed up to zero.
Compatibility is essentially a statement of how the structure must fit together.
Compatibility is the relation between the deformations of the system.

3.4 FORCE DISPLACEMENT RELATION:

In the study of deformed bodies there is a term constitutive law which refers
to the relations between stresses and strain.Using the Constitutive laws for a given material and the concepts of equilibrium and compatibility we can define the force deformation relation of any structural element.
There are two basic ways to express the relations.the first way to express the relation is

$$F=ke.$$

Where f and e are member force and displacement.
K is the stiffness of the element.
The stiffness has units of force per length and may be thought of as force necessary to hold the element to a unit

displacement. the second form of force displacement equation is

$$e=1/k*f$$
$$e=Ff$$

in this case the quantity f has units of length per force and defines the flexibility of the structural element. A flexibility coefficient may be thought of as the displacement that results from a unit load. We can see that in this model the flexibility is simply reciprocal of the stiffness. the flexibilities and stiffnesses of elements with multiple element forces are also related by this inverse property.

3.5 STEPS TO BASIC STIFFNESS PROPERTIES:

The steps of the basic stiffness method can be stated as follows

1. P =f(F), e=f(d), F=f(e).
2. e=f(d),F=f(e)=f(d).
   Substitute the compatibility relations into force displacement relation to obtain a system of equation of equations relating the member forces to the displacement.
3. F=f(d), p=f(F)=f(d).
   Substitute the member forces now in terms of d, into the equilibrium equation to obtain a system of equations relating the structural forces to the structural displacement.
4. Solve the equations of step 3, which are written in the directions of the degrees of freedom for free displacement.
5. solve the equations in step three that are written in the directions of the prescribed displacement for the forces in the directions of the prescribed displacements.
6. step 4 completely defines displacements. These displacements may now be substituted to the equations of step 2 to determine the member forces.

3.6 THE USE OF MATRICES:

We have already seen the use of condition of equilibrium; compatibility and force displacement are all expressed in terms of systems of simultaneous equations. Through series of substitutions it is possible to form a system of n equations in n unknowns that has a solution. The substitution process and eventual solution can be more conveniently expressed in a matrix format. The matrix operations are essentially an efficient way to carry out the substitution process among different sets of simultaneous equation. In addition to convenience and compactness for hand computations, matrices are conveniently represented by arrays and vectors in computer codes. Once the steps of a method are described in a matrix format the transfer to computer code is reliably straightforward.

So below are the some general notations for the matrices that will be used in each of the steps of basic stiffness method.

1. Equilibrium, compatibility and force displacement relations. We have developed the idea of statics matrix [B] that relates applied and reactive forces. Using the notations, our equilibrium equations in the directions of the steps of the basic stiffness method.

$$\{P\}=[B]\{F\}$$

P is the applied loads.
B statics matrix.
F spring forces.
The compatibility relations connects the internal deformations {e} to the free motions of the joints{d}

$$\{e\}=[A]\{d\}$$

e spring elongations.
A compatibility matrix.
D degrees of freedom.
Finally the load displacement relations for the stiffness method are in the form F=ke.
So in the matrix form we can write

$$\{F\}=[k]\{e\}$$

Where F denotes spring forces.
K is unassembled stiffness matrix.
e is spring deformations.

With these definitions we can now express the remaining steps symbolically.

2. Substituting the compatibility relation into the force displacement relation.

$$\{e\}=[A]\{d\}$$
$$\{F\}=[K]\{e\}$$
$$\{F\}=[K][A]\{d\}$$

3. Now substituting {F} into equilibrium condition.

$$\{F\}=[k][A]\{d\}$$
$$\{P\}=[B]\{F\}$$
$$\{P\}=[B][k][A]\{d\}$$

Now here we can combine the triplet product [B][K][A] into one matrix to obtain

$$\{P\}=[K]\{d\}$$

Where [K] is structure stiffness matrix.

Now solving the equation for the displacement d

$$\{d\}=\{P\}/[K]$$

5. Now returning to the result of step 2 and compute {F} from the equation

$$\{F\}=[k][A]\{d\}$$

Above all the matrices formed it is a straightforward set of matrix operations that yields a complete solution for all displacements and all forces.

3.7
INTRODUCTION TO FINITE ELEMENT METHOD:

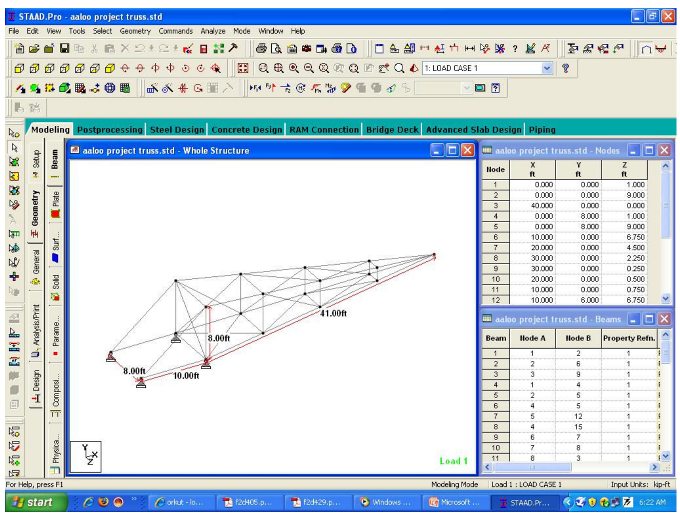
After having the idea of matrices applications we can start the finite element method. In a truss, every element is in direct compression and tension. Classical methods like methods of sections, method of joints are still in use but these methods are tedious when applied to large scale statically indeterminate structure. Further joints displacements are not readily obtainable. The finite element method method on the other hand is applicable to statically determinate or indeterminate structure. The finite element method also provides the joint deflections. Effects of temperature changes and support settlements are also routinely handled. Elements of a truss have various orientations. To account for these different orientations local and global co-ordinate systems are introduced.

To assist these co ordinate systems, simultaneously local and global stiffness matrix is also introduced in finite element method.

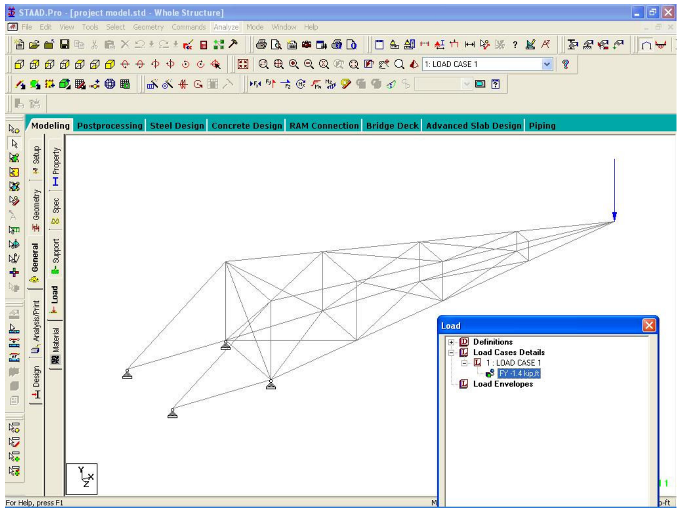
Global stiffness matrix is formed by the superposition of local stiffness matrixes.

Finally stiffness matrix is the integral part of force displacement equation and subsequently we can venture into some solutions like stress calculations, joint displacement, temperature effects etc.

So we can solve statically determinate or indeterminate truss alike in finite element methods. It has edge over the classical methods.



4.1 A truss undergoing point load at edge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File Edit View Tools Select Geometry Commands Analyze Mode Window Help
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Modeling
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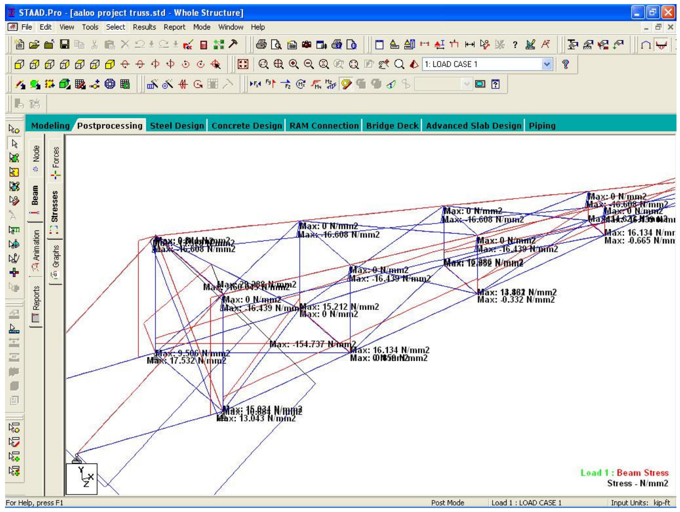


4.2 Member stresses

5 DYNAMIC ANALYSIS

All real physical structures, when subjected to loads or displacements, behave dynamically. The additional inertia forces, ***from Newton's second law***, are equal to the mass times the acceleration. If the loads or displacements are applied very slowly then the inertia forces can be neglected and a static load analysis can be justified. Hence, dynamic analysis is a simple extension of static analysis.
In addition, all real structures potentially have an infinite number of displacements. Therefore, the most critical phase of a structural analysis is to create a computer model, with a finite number of massless members and a finite number of node (joint) displacements that will simulate the behaviour of the real structure. The mass of a structural system, which can be accurately estimated, is lumped at the nodes. Also, for linear elastic structures the stiffness properties of the members, with the aid of experimental data, can be approximated with a high degree of confidence. However, the dynamic loading, energy dissipation properties and boundary (foundation) conditions for many structures are difficult to estimate. This is always true for the cases of seismic input or wind loads. To reduce the errors that may be caused by the approximations summarized in the previous paragraph, it is necessary to conduct many different dynamic analyses using different computer models, loading and boundary conditions. It is not realistic to conduct 20 or more computer runs to design a new structure or to

5.1
STATIC AND DYNAMIC ANALYSIS

Because of the large number of computer runs required for a typical dynamic analysis, it is very important that accurate and numerically efficient methods be used within computer programs.

5.2
DYNAMIC EQUILIBRIUM

The force equilibrium of a multi-degree-of-freedom lumped mass system as a function of time can be expressed by the following relationship:

$\mathbf{F}(t)_I + \mathbf{F}(t)_D + \mathbf{F}(t)_S = \mathbf{F}(t)$ in which the force vectors at time t are $\mathbf{F}(t)_I$ is a vector of inertia forces acting on the node masses

$\mathbf{F}(t)_D$ is a vector of viscous damping, or energy dissipation, forces

$\mathbf{F}(t)_S$ is a vector of internal forces carried by the structure $\mathbf{F}(t)$ is a vector of externally applied loads

Equation is based on physical laws and is valid for both linear and nonlinear systems if equilibrium is formulated with respect to the deformed geometry of the structure. For many structural systems, the approximation of linear structural behavior is made in order to convert the physical equilibrium statement, Equation to the following set of second-order, linear, differential equations:

$\mathbf{M}\mathbf{u}(t)_a + \mathbf{C}\mathbf{u}(t)_a + \mathbf{K}\mathbf{u}(t)_a = \mathbf{F}(t)$

in which $\mathbf{M}$ is the mass matrix (lumped or consistent), $\mathbf{C}$ is a viscous damping matrix (which is normally selected to approximate energy dissipation in the real structure) and $\mathbf{K}$ is the static stiffness matrix for the system of structural elements. The time-dependent vectors $\mathbf{u}(t)_a$, $\&\mathbf{u}(t)_a$ and $\&\&\ \mathbf{u}(t)_a$ are the absolute node displacements, velocities and accelerations, respectively. Investigate retrofit options for an existing structure. For seismic loading, the external loading $\mathbf{F}(t)$ is equal to zero. The basic seismic motions are the three components of free-field ground displacements $u(t)_{ig}$ that are Known at some point below the foundation level of the structure. Therefore, we can write Equation (2) in terms of the displacements $\mathbf{u}(t)$, velocities $\&\mathbf{u}(t)$ and accelerations $\&\&\ \mathbf{u}(t)$ that are relative to the three components of free-field ground displacements. Therefore, the absolute displacements, velocities and accelerations can be eliminated from Equation by writing the following simple equations:

$$\mathbf{u}(t)a = \mathbf{u}(t) + \mathbf{I}\, u(t)xg + \mathbf{I}\, u(t)yg + \mathbf{I}\, u(t\, zg).$$

where $\mathbf{I}i$ is a vector with ones in the "*i*" directional degrees-of-freedom and zero in all other positions. The substitution of Equation (3) into Equation (2) allows the node point equilibrium equations to be rewritten as

$$\mathbf{Mu}(t) + \mathbf{Cu}(t) + \mathbf{Ku}(t) = -\mathbf{M}\, xu(t)xg - \mathbf{M}\, yu(t)yg - \mathbf{M}zu(t)zg$$

The simplified form of Equation (4) is possible since the rigid body velocities and displacements associated with the base motions cause no additional damping or structural forces to be developed.

It is important for engineers to realize that the displacements, which are normally printed by a computer program, are relative displacements and that the fundamental loading on the structure is foundation displacements and not externally applied loads at the joints of the structure. For example, the static pushover analysis of a structure is a poor approximation of the dynamic behaviour of a three dimensional structure subjected to complex time-dependent base motions. Also, one must calculate absolute displacements to properly evaluate base isolation systems. There are several different classical methods that can be used for the solution of Equation. Each method has advantages and disadvantages that depend on the type of structure and loading. To provide a general background for the various topics presented in this book, the different numerical solution methods are summarized below.

5.3
STEP BY STEP SOLUTION METHOD

The most general solution method for dynamic analysis is an incremental method in which the equilibrium equations are solved at times D*t*, 2D*t*, 3D*t*, etc. There are a large number of different incremental solution methods. In general, they involve a solution of the complete set of equilibrium equations at each time increment. In the case of nonlinear analysis, it may be necessary to reform the stiffness matrix for the complete structural system for each time step. Also, iteration may be required within each time increment to satisfy equilibrium. As a result of the large computational requirements it can take a significant amount of time to solve structural systems with just a few hundred degrees-of-freedom. In addition, artificial or numerical damping must be added to most incremental solution methods in order to obtain stable solutions. For this reason, engineers must be very careful in the interpretation of the results. For some nonlinear structures, subjected to seismic motions, incremental solution methods are necessary.

For very large structural systems, a combination of mode superposition and incremental methods has been found to be efficient for systems with a small number of nonlinear members. This method has been incorporated in the new versions of SAP and ETABS.

5.4 MODE SUPERPOSITION METHOD

The most common and effective approach for seismic analysis of linear structural systems is the mode superposition method. This method, after a set of orthogonal vectors are evaluated, reduces the large set of global equilibrium equations to a relatively small number of uncoupled second order differential equations. The numerical solution of these equations involves greatly reduced computational time. It has been shown that seismic motions excite only the lower frequencies of the structure. Typically, earthquake ground accelerations are recorded at increments of 200 points per second. Therefore, the basic loading data does not contain information over 50 cycles per second. Hence, neglecting the higher frequencies and mode shapes of the system normally does not introduce errors.

5.5 RESPONSE SPECTRA ANALYSIS

The basic mode superposition method, which is restricted to linearly elastic analysis, produces the complete time history response of joint displacements and member forces due to a specific ground motion loading [1,2]. There are two major disadvantages of using this approach. First, the method produces a large amount of output information that can require

an enormous amount of computational effort to conduct all possible design checks as a function of time. Second, the analysis must be repeated for several different earthquake motions in order to assure that all the significant modes are excited, since a response spectrum for one earthquake, in a specified direction, is not a smooth function. There are significant computational advantages in using the response spectra method of seismic analysis for prediction of displacements and member forces in structural
systems. The method involves the calculation of only the maximum values of the displacements and member forces in each mode using smooth design spectra that are the average of several earthquake motions. In this book, we will recommend the CQC method to combine these maximum modal response values to obtain the most probable peak value of displacement or force. In addition, it will be shown that the SRSS and CQC3 methods of combining results from orthogonal earthquake motions will allow one dynamic analysis to produce design forces for all members in the structure.

5.6 SOLUTION IN THE FREQUENCY DOMAIN

The basic approach, used to solve the dynamic equilibrium equations in the frequency domain, is to expand the external loads **F***(t)* in terms of Fourier series or Fourier integrals. The solution is in terms of complex numbers that cover the time
span from -¥ to ¥. Therefore, it is very effective for periodic types of loads such as mechanical vibrations, acoustics, sea-waves and wind [1]. However, the use of the frequency domain solution method for solving structures subjected to earthquake motions has the following disadvantages:

1. The mathematics, for most structural engineers including myself, is difficult to understand. Also, the solutions are difficult to verify.

2. Earthquake loading is not periodic; therefore, it is necessary to select a long time

period in order that the solution from a finite length earthquake is completely damped out prior to the application of the same earthquake at the start of the next period of loading.

3. For seismic type loading the method is not numerically efficient. The transformation of the result from the frequency domain to the time domain, even with the use of Fast Fourier Transformation methods, requires a significant amount of computational effort.

4. The method is restricted to the solution of linear structural systems.

5. The method has been used, without sufficient theoretical justification, for the approximate nonlinear solution of site response problems and soil/structure interaction problems. Typically, it is used in an iterative manner to create linear equations. The linear damping terms are changed after each iteration in order to approximate the energy dissipation in the soil. Hence, dynamic equilibrium,within the soil, is not satisfied.

The step-by-step solution of the dynamic equilibrium equations, the solution in the frequency domain, and the evaluation of eigenvectors and Ritz vectors all require the solution of linear equations of the following form:

AX = **B** Where **A** is an 'N by N' symmetric matrix which contains a large number of zero terms. The 'N by M' **X** displacement and **B** load matrices indicate that more than one load condition can be solved at the same time. The method used in many computer programs, including SAP2000 [5] and ETABS [6], is based on the profile or active column method of compact storage. Because the matrix is symmetric, it is only necessary to form and store the first nonzero term in each column down to the diagonal term in that column. Therefore, the sparse square matrix can be stored as a one dimensional array along with a ***N by 1*** integer array that ndicates the location of each diagonal term. If the stiffness matrix exceeds the high-speed memory capacity of the computer a block storage form of the algorithm exists. Therefore, the capacity of the solution method is governed by the low speed disk capacity of the computer. This solution method is presented in detail in Appendix C of this book.

5.7
UNDAMPED HARMONIC RESPONSE

The most common and very simple type of dynamic loading is the application of steady-state harmonic loads of the following form:

F*(t)* **= f** sin*(*w *t)* The node point distribution of all static load patterns, **f** , which are not a function of time, and the frequency of the applied loading, w , are user specified. Therefore, for the case of zero damping, the exact node point equilibrium equations for the structural system are

Mu*(t)* + **Ku***(t)*= **f** sin*(*w *t)* The exact steady-state solution of this equation requires that the node pointdisplacements and accelerations are given by

u(t) = **v** sin*(t) ,* Therefore, the harmonic node point response amplitude is given by the solution of the following set of linear equations:

Kv = **f**

It is of interest to note that the normal solution for static loads is nothing more than a solution of this equation for zero frequency for all loads. It is apparent that the computational effort required for the calculation of undamped steady-state response
is almost identical to that required by a static load analysis. Note that it is not necessary to evaluate mode shapes or frequencies to solve for this very common type of loading. The resulting node point displacements and member forces vary as sin*(*w *t)*. However, other types of loads that do not

vary with time, such as dead loads, must be evaluated in a separate computer run.

5.8
UNDAMPED FREE VIBRATIONS

Most structures are in a continuous state of dynamic motion because of random loading such as wind, vibrating equipment, or human loads. These small ambient vibrations are normally near the natural frequencies of the structure and are terminated by energy dissipation in the real structure. However, special instruments attached to the structure can easily measure the motion. Ambient vibration field
tests are often used to calibrate computer models of structures and their foundations.After all external loads are removed from the structure, the equilibrium equation, which governs the undamped free vibration of a typical displaced shape **v,** is

Mv + **Kv** = 0

At any time the displaced shape **v** may be a natural mode shape of the system, or
any combination of the natural mode shapes. However, it is apparent the total energy within an undamped free vibrating system is a constant with respect to time. The sum of the kinetic energy and strain energy, at all points in time, is a constant
and is defined as the ***mechanical energy*** of the dynamic system and can be calculated from:

E(m)=1/2v(t)Mv+1/2v(t)Kv.

Dynamic analysis of three dimensional structural systems is a direct extension of static analysis. The elastic stiffness matrices are the same for both dynamic and static analysis. It is only necessary to lump the mass of the structure at the joints.The addition of inertia forces and energy dissipation forces will satisfy dynamic equilibrium. The dynamic solution for steady state harmonic loading, without damping, involves

the same numerical effort as a static solution. Classically, there
are many different mathematical methods to solve the dynamic equilibrium equations. Energy is fundamental in dynamic analysis. At any point in time the external work supplied to the system must be equal to the sum of the kinetic and strain energy plus the energy dissipated in the system.It is my opinion, with respect to earthquake resistant design, that we should try tominimize the mechanical energy in the structure. It is apparent that a rigid structure will have only kinetic energy and zero strain energy. On the other hand, a completely base isolated structure will have zero kinetic energy and zero strain energy. A structure cannot fail if it has zero strain energy.

BUILDING UNDER SEISMIC LOAD

6.1
Theoretical background:

Any type of movement in the earth surface is known as earthquake. It may be caused by natural or man-made activities. For design purposes ground motions are derived from strong motion accelerograms that are recorded by special accelerograph instruments. The velocity and displacement may be found by integrating acceleration.

There are several methods for finding the duration of shaking. The summation method Trifunac and Brady is a famous one (1975).

6.2
Response spectra:

A response spectrum is a plot of the peak values of the response (displacement, velocity, or acceleration) of a number of SDOF systems with different natural vibration periods subjected to the same seismic input. Therefore, an acceleration response spectrum represents the peak accelerations that a suite of SDOF systems with a range of natural periods may exhibit when subject to a given ground motion component.

Site-specific response spectra are developed using source to site distances, appropriate attenuation relationships, expected magnitudes, and actual local site conditions. Therefore, it is typically assumed that site-specific studies will provide more accurate acceleration spectra than using the codified standard acceleration spectra. Headquarters, U.S. Army Corps of Engineers (1999) describes the conditions requiring a site-specific ground motion study. Site-specific response spectra can be generated by means of a deterministic seismic hazard analysis (DSHA) or a probabilistic seismic hazard analysis

(PSHA). In the DSHA, the site ground motions are estimated for a specific earthquake scenario, defined as a seismic event of a certain magnitude for a particular seismic source occurring at a certain distance from the site. The representation of the ground motions in terms of the corresponding site-specific response spectra is achieved by using appropriate attenuation relationships. Information on this approach can be found in HQUSACE (1999). The PSHA is an approach that uses the likelihood (probability) that a given level of ground motion will occur during a specific exposure period. In the PSHA, the site ground motions are defined for selected values of
the probability of exceedance in a given time exposure period, or for selected values of annual frequency or return period for ground motion exceedance.

6.3
Seismic zones in india:

The varying geology at different locations in the country implies that the likelihood of damaging earthquakes taking place at different locations is different. Thus, a seismic zone map is required to identify these regions. Based on the levels of intensities sustained during damaging past earthquakes, the 1970 version of the zone map subdivided India into five
zones – I, II, III, IV and V (Figure 3). The maximum Modified Mercalli (MM) intensity of seismic shaking expected in these zones were *V or less*, *VI*, *VII*, *VIII*, and *IX and higher*, respectively.

6.4

Seismic coefficients:

The Seismic Coefficients are dimensionless coefficients which represent the (maximum) earthquake acceleration as a fraction of the acceleration due to gravity. Typical values are in the range of 0.1 to 0.3.

When a seismic coefficient is defined, an additional Body Force will be applied to *each finite element in the mesh*, as follows:

Seismic Force = Seismic Coefficient * Body Force (due to gravity)

= Seismic Coefficient * (area of element * Unit Weight of element material)

- Body Force (due to gravity) is simply the self-weight of a finite element.
- The Seismic body force is vectorially added to the (downward) Body Force which exists due to gravity, to obtain the total body force acting on the element.

6.5

Seismic zone factor:

The seismic zone factor (or Z factor) corresponds numerically to the effective horizontal peak bedrock acceleration (or equivalent velocity) that is estimated as a component of the design base shear calculation. For instance, the area within seismic Zone 1 (Z-factor of 0.1) should expect an earthquake-related effective peak bedrock acceleration of 0.1 times the force of gravity. These values correspond to ground motion values with a 10 percent probability of being exceeded in 50 years.

Seismic Zone 2 is subdivided into two regions. Seismic Zone 2A has a Z-factor of 0.15 and is not associated with a particular fault zone: Seismic Zone 2B (not in this mapping area) has a Z factor of 0.20 and indicates an association with known crustal faults.

6.6

Structure importance factor:

Most building codes over the world require that very important structures be designed for a seismic coefficient equal to that used for ordinary structures multiplied by a factor greater than one, called the importance factor. This factor is set intuitively or arbitrarily, varying between very wide limits and always independent of the design coefficients of ordinary structures; in other words, independent of the site seismicity and the properties of the structures. Thus, the International Building Code stipulates factors of 1 or 1.5 depending on the type of facility.

6.7

Criteria for designing building under seismic loading:

Earthquakes cause random motion of ground which can be resolved in any three mutually perpendicular directions. This motion causes the structure to vibrate. The vibration intensity of ground expected at any location depends upon the magnitude of earthquake, the depth of focus, distance from the epicentre and the strata on which the structure stands.

The predominant direction of vibration is horizontal. Relevant combinations of forces applicable for design of a particular structure have been specified in the relevant clauses.

3.1.2 The response of the structure to the ground vibration is a function of the nature of foundation soil; materials, form, size and mode of construction of the struture; and the duration and the intensity of ground motion. This standard specifies design seismic coefficient for structures standing on soils or rocks which will not settle or slide due to Ioss of strength during vibrations.

6.8

IS: 1893 – 1984 SPECIFICATION:

The seismic coefficients recommended in this standard are based on design practice conventionally followed and performance of structures in past earthquakes, It is well understood that the forces which structures would be subjected to in actual earthquakes, would be very much larger than specified in this standard as basic seismic coefficient. In order to take care of this gap, for special cases importance factor and performance factor
(where necessary) are specified in this standard elsewhere.3.1.4 In the case of structures designed for horizontal seismic force only, it shall be considered to act in any one direction at a time. Where both horizontal and vertical seismic forces are taken into account, horizontal
force in any one direction at a time may be considered simultaneously with the vertical force as specified in 3.4.5. 3.1.5 The vertical seismic coefficient shall be considered in the case of
structures in which stability is a criterion of design or, for overall stability, analysis of structures except as otherwise stated in the relevant clauses. 3.1.6 Equipment and systems supported at various floor levels of structures will be subjected to motions corresponding to vibrations at their support points. In important cases, it may be necessary to obtain floor
response spectra for design.

Assumptions - The following assumptions shall be made in the earthquake resistant design of structures:

1) Earthquake causes impulsive ground motion which is complex and irregular in character, changing in period and amplitude each lasting for small duration. 'Therefore, resonance of the type as visualized under steady state sinusoidal excitations will not occur as it would need time to build up such amplitudes.

2) Earthquake is not likely to occur simultaneously with wind or maximum flood or Maximum sea waves.

3) The value of elastic modulus of materials, wherever required, may be taken as for static analysis unless a more definite value is available for use in such condition.

6.9
Permissible Increase in Stresses and Load Factors

Whenever earthquake forces are considered along with other normal design forces, the permissible stresses in materials, in the elastic method of design, may be increased by one-third. However, for steels having a definite yield stress, the stress be limited to the yield stress; for steels without a definite yield point, the will stress will be limited to 80 percent of the ultimate strength or 0.2 percent proof stress whichever is smaller and that in prestressed concrete members, the tensile stress in the extreme fibres of the concrete may be permitted so as not to exceed 213 of the modulus of rupture of concrete.

Load Factors - Whenever earthquake forces are considered along with other normal design forces, the following factors may be adopted:

a) For ultimate load design of steel structures:
UL = 1*4(DL+LL+EL) where

UL = the ultimate load for which the structure or its elements should be designed according to the relevant Indian Standards for steel structures;

DL = the dead load of the structure;

LL = the superimposed load on the structure considering its Modified values as given in the relevant clauses of this Standard

EL = the value of the earthquake load adopted for design,

For limit state design of reinforced and prestressed concrete Structures.

The partial safety factors for limit states of serviceability **and** collapse and the procedure for design as given in relevant Indian Standards (ste IS : 456-1978* and IS *: 1343-1980t)* ‘may be used for earthquake loads combined with other normal loads, The live load values to be used shall be as given in the relevant clauses of this standard.

6.10
<u>Design Criteria for Multi-storeyed Buildings:</u>

A) The criteria for design of multi-storeyed buildings &ail be as In case of buildings with floors capable of providing rigid horizontal diaphragm action, a separate building or any block of a building between two separation sections shall be analyzed as a whole for seismic forces .The total shear in any horizontal plane shall be distributed to various elements of lateral forces resisting system assuming the floors to be infinitely rigid in the horizontal plane, In buildings having shear walls together with frames, the frames shall be designed for at least 25 percent of the seismic shear.

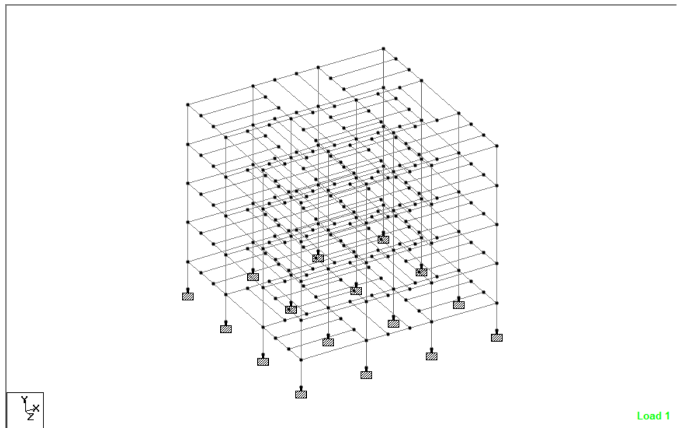
B) In case of buildings where floors are not able to provide the diaphragm action as independently; in (a) above the building frames behave and may be analyzed frame by frame with tributary masses for seismic forces.

C) If the building height is above 90 meters and it is in the seismic zone one or two response spectra method is to be applied.

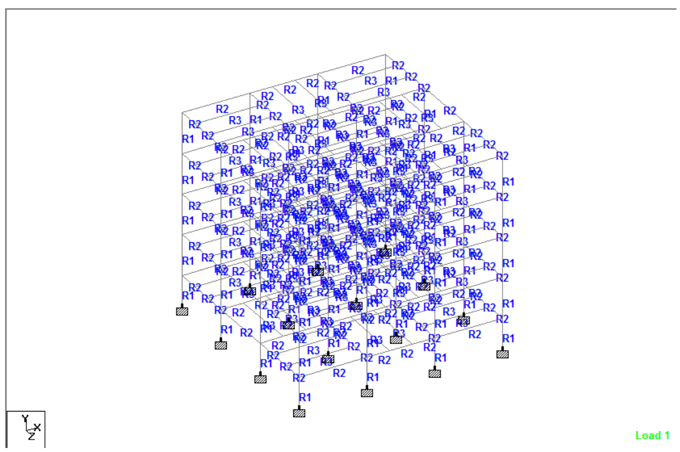
If the building height is between 40 to 90 meters and is in all zones response spectra and seismic coefficient method may be applied.

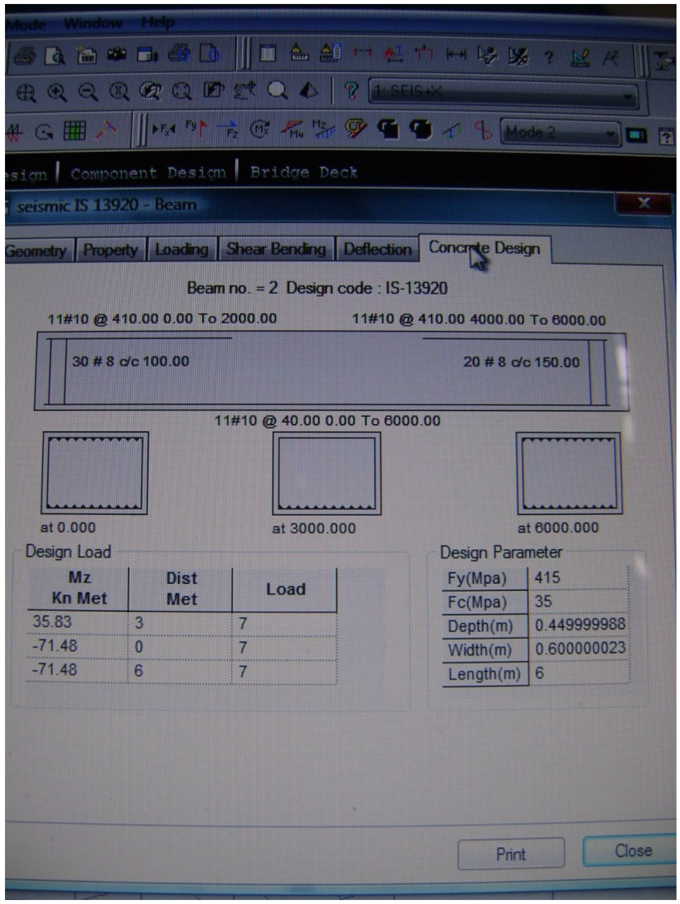
If the building height is below 40 meters and is in all seismic zones seismic coefficient method may be applied.

FRAME OF THE BUILDING



DIFFERENT MEMBERS





ONE DESIGN DETAILS FOR A BEAM

8. CONCLUSION

From the above study and results it has been found that Staad pro is very useful in civil engineering design and analysis purposes. There may be some minor errors and warnings which may be evaded by the application of more practical data and skilfully taken input value and commands.

9. REFERENCES:

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